Golder Associates

FOUNDATION INVESTIGATION AND DESIGN REPORT DIVISION STREET OVERPASS WIDENING HIGHWAY 401 WIDENING FROM WEST OF SYDENHAM ROAD TO WEST OF MONTREAL STREET KINGSTON, ONTARIO W.P. 77-99-01

Submitted to:

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SYDENHAM ROAD TO WEST OF MONTREAL STREET
KINGSTON, ONTARIO
W.P. 77-99-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the Highway 401 widening from four to six lanes, from west of Sydenham Road to west of Montreal Street in the City of Kingston, Ontario. Foundation engineering services are required for the following components under W.P. 77-99-01:

- northward widening of the existing Division Street overpass structure;
- investigation of instability and settlement along a section of the Division Street W-N/S Ramp;
- widening of the high fill embankment in the vicinity of Little Cataraqui Creek, between Sydenham Road and Sir John A. MacDonald Boulevard;
- overhead signs; and
- trenchless sewer installation.

This report addresses the northward widening of the Division Street overpass structure, including northward widening of its approach embankments.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated January 2005, and in Section 6.8 of MRC's *Technical Proposal* for this project.

2.0 SITE DESCRIPTION

The Division Street overpass structure site is located on Highway 401 in the City of Kingston, in Frontenac County. The existing Division Street overpass, which was constructed in the 1950s, is a single-span structure supported on timber piles.

The natural ground surface at the structure site is relatively flat, at about Elevation 82 m to 83 m. Division Street has been constructed close to the original ground surface, with its grade in the vicinity of Highway 401 at about Elevation 82.5 m. The Highway 401 grade at the existing overpass structure is at about Elevation 88.5 m to 89 m (rising eastward). The existing west and east approach embankments are approximately 6 m to 6.5 m high relative to the surrounding natural ground surface. To the north and west of the Highway 401- Division Street interchange, a bedrock outcrop is present; the ground surface elevation in the outcrop area is up to several metres higher than that in the vicinity of the existing overpass structure.

Observations of the existing Division Street overpass structure and adjacent approach embankments (including side slopes) by senior Golder personnel indicate that there does not appear to be any settlement-related distress to the existing structure or embankments.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in February 2006, at which time a total of four boreholes (Boreholes 06-1 to 06-4) were advanced using a CME 55 track-mounted drill rig, supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. In January 2007, two additional boreholes (Boreholes 07-21 and 07-22) were advanced, using a CME 75 track-mounted drill rig, through the Highway 401 embankment fill in the vicinity of the Division Street overpass to provide subsoil information related to potential temporary roadway protection systems, if required.

The boreholes were advanced at the locations shown on Drawing 1. Boreholes 06-2 and 06-3 were advanced as close as practicable to the west and east abutment widening areas, respectively; these boreholes were drilled to total depths of 16.8 m and 14.6 m, including more than 3 m of bedrock coring. Boreholes 06-1 and 06-4 were advanced within the limits of the west and east approach embankment widening, respectively, to a depth of 6.7 m. Boreholes 07-21 and 07-22 were advanced through the existing highway embankment to the west and east of the existing overpass structure, to depths of 8.1 m and 6.6 m, respectively.

Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In situ vane testing, using an MTO "N"-size vane, was carried out to measure the undrained shear strength of the firm to stiff portions of the silty clay to clay deposit that was encountered at the site. The bedrock coring in Boreholes 06-2 and 06-3 was completed using an "NQ"-size rock core barrel.

A standpipe piezometer was installed in Borehole 06-1, within the clay deposit. The piezometer consists of 25 mm diameter PVC pipe with a slotted tip installed within a 1.5 m thick filter sand pack. A 0.3 m thick bentonite seal was placed on top of the filter sand, followed by a mixture of bentonite and clay soil to the ground surface, where a 0.3 m thick bentonite seal was placed around the piezometer casing. The remaining boreholes were backfilled to the ground surface using bentonite, in places mixed with native clayey soil (cuttings from the borehole), in accordance with Ontario Regulation 128 (amendment to Ontario Regulation 903).

The field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and ground surface elevations were provided by J.D. Barnes Surveying Ltd. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	*Borehole Location*	*MTM NAD83 Northing (m)*	*MTM NAD83 Easting (m)*	*Ground Surface Elevation (m)*
06-1	West approach	4,903,276.1	304,850.0	81.9
06-2	West abutment	4,903,268.1	304,862.0	82.7
06-3	East abutment	4,903,262.9	304,882.6	82.5
06-4	East approach	4,903,269.7	304,894.7	82.8
07-21	West embankment	4,903,260.3	304,822.3	88.0
07-22	East embankment	4,903,249.0	304,936.8	89.5

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the southern portion of the physiographic region of Southern Ontario known as the Napanee Plain, as delineated in *The Physiography of Southern Ontario*[1]. The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping[2] indicates that the bedrock within the Napanee Plain consists of grey limestone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay. Well records indicate that the average depth to bedrock within the Napanee Plain is approximately 2 m. However, in many areas, bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the southern portion of the Napanee Plain, and within and adjacent to river valleys throughout the Plain.

4.2 Site Stratigraphy

As part of the subsurface investigation at the Division Street overpass site, four boreholes were advanced within the widening area for the abutments and approach embankments, and two boreholes were advanced through the Highway 401 embankment fill to provide subsoil information related to potential temporary roadway protection systems, if required. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 to 6 following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil and bedrock conditions will vary between and beyond the borehole locations.

In summary, the soils encountered within the overpass widening area consist of topsoil or fill overlying an approximately 1 m to 1.9 m thick surficial deposit that varies in composition from a clayey silt to clay containing organics, to an organic silt. The surficial soils are underlain by a clay deposit, which is in turn underlain by a relatively thin till deposit. The overburden soils are

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[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario.* Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

[2] Map 2544, Ministry of Northern Development and Mines, 1991.

underlain by limestone bedrock, which was encountered in the boreholes between about 11.5 m and 13.4 m depth (at about Elevation 71.0 m and 69.3 m, respectively).

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil / Fill

About 200 mm of topsoil was encountered in Boreholes 06-1 and 06-4, which were located in the grassed area at the north toe of the existing west and east approach embankments.

Approximately 600 mm to 800 mm of fill (comprised of silty sand or gravel containing silty organics) was encountered in Boreholes 06-2 and 06-3, which were located at the north toe of the existing Highway 401 embankment, immediately west and east of the Division Street shoulder.

The two boreholes advanced through the existing Highway 401 WBL embankment to the west and east of the overpass structure (Boreholes 07-21 and 07-22, respectively) penetrated 7.5 m and 5.9 m of heterogeneous fill and terminated within the underlying silty clay. The embankment fill encountered in these boreholes can be classified into three major zones:

- an upper zone consisting of layered sand and gravel to sand to clayey silt;
- a middle zone of gravel and cobble-sized rock fill; and
- a lower zone of clayey silt (see grain size distribution test results from two selected samples on Figure 1), which contains a thin lens of topsoil at one location.

The upper zone of embankment fill is generally compact but has very dense or soft to very stiff layers, with SPT "N" values ranging from 3 to 79 blows per 0.3 m of penetration. The rock fill layer is loose to very dense, with SPT "N" values ranging from 4 to 47 blows per 0.3 m of penetration and up to 50 blows per 0.01 m of penetration due to the presence of a cobble/boulder. The lower layer of clayey silt fill is stiff to very stiff in consistency, with SPT "N" values ranging from 10 to 16 blows per 0.3 m of penetration.

4.2.2 Surficial Clayey Silt to Clay and Organic Silt

A 1.0 m to 1.9 m thick layer of surficial soil was encountered directly below the topsoil and/or fill at this site; the base of this surficial deposit was encountered in the boreholes between Elevation 80.2 m and 80.8 m.

The surficial deposit varies in composition from clayey silt to clay containing organics, to an organic silt. Organic content testing was carried out on two samples of the clayey silt to clay, and measured organic contents of 6.6 and 6.8 per cent. An Atterberg limits test was carried out on one sample of the surficial deposit, and measured a plastic limit of 38 per cent, a liquid limit of 73 per cent, and a corresponding plasticity index of 35 per cent; this result, which is plotted on a plasticity chart on Figure 2, confirms that the material encountered in Borehole 06-4 is an organic silt.

The measured SPT "N" values within this material vary from 2 to 10 blows per 0.3 m of penetration but are typically less than 5 blows per 0.3 m of penetration, indicating that the material has a soft to stiff, but typically soft to firm, consistency.

4.2.3 Clay to Clayey Silt

The topsoil, fill and surficial soils are underlain by a typically grey clay deposit; the deposit grades to a clayey silt with depth in one of the boreholes. The surface of the clay deposit was encountered between Elevations 80.2 m and 80.8 m within the footprint of the overpass/approach embankment widening, and as high as Elevation 83.6 m in Borehole 07-22. The base of the deposit, where fully penetrated in Boreholes 06-2 and 06-3, was encountered between Elevations 71.1 m and 74.0 m. The deposit is therefore approximately 6.7 m and 9.1 m thick in the two boreholes where it was fully penetrated.

The results of grain size distribution tests carried out on two selected samples of the clay deposit are provided on Figure 3. Atterberg limits testing was carried out on ten samples of this deposit. In general, the measured plastic limits vary from 21 to 26 per cent, the liquid limits from 45 to 63 per cent, and the plasticity indices from 23 to 38 per cent; however, for one sample from near the base of the deposit in Borehole 06-3, the Atterberg limits testing measured a plastic limit of 18 per cent, a liquid limit of 33 per cent, and a plasticity index of 15 per cent. These results, which are plotted on a plasticity chart on Figure 4, confirm that the deposit generally consists of a high plasticity clay, which grades to a clayey silt of low plasticity as encountered near the bottom of the deposit in Borehole 06-3.

The measured SPT "N" values within the clay to clayey silt range from 2 to 12 blows per 0.3 m of penetration. In situ vane testing was carried out within the clay in each of the four boreholes, typically measuring an undrained shear strength ranging from 80 kPa to greater than 100 kPa; however, lower undrained shear strengths of 38 to 42 kPa were measured in the upper portion of the clay deposit as encountered in Borehole 06-2. The vane test results indicate that the grey clay generally has a stiff to very stiff consistency, except for the upper portion of the clay encountered between about Elevation 78 m and 80 m in Borehole 06-2 which has a firm consistency.

Where undrained shear strength of less than 100 kPa were measured, remoulded shear strengths were also measured in order to determine the sensitivity of the clay deposit. Based on these results, the sensitivity of the clay varies from approximately 2.5 to 4.7; these results indicate that the clay has a medium sensitivity (according to the *Canadian Foundation Engineering Manual*).

4.2.4 Clayey Silt Till to Silty Sand Till

A 1.8 m to 3.0 m thick layer of till was encountered below the clay to clayey silt deposit in Boreholes 06-2 and 06-3. The surface of this till deposit was encountered at about Elevations 71.1 m and 74.0 m in the boreholes (at depths below ground surface of 11.6 m and 8.5 m, respectively).

The till varies in composition from clayey silt with sand, some gravel to silty sand, some gravel trace clay. The result of a grain size distribution test completed on one selected sample of the till is provided on Figure 5. An Atterberg limits test was carried out on one sample of the till, and measured a plastic limit of 4 per cent, a liquid limit of 14 per cent, and a corresponding plasticity index of 10 per cent. This result, which is plotted on a plasticity chart on Figure 6, indicates that the till is a clayey silt of low plasticity, bordering on a non-plastic (i.e. silty sand, trace clay) material.

The measured SPT "N" values within the thin till layer range from 19 to 72 blows per 0.3 m of penetration, indicating that the till has a very stiff to hard consistency / compact to very dense relative density.

4.2.5 Limestone Bedrock

Limestone bedrock underlies the till deposit at this site. The following table summarizes the depth to the bedrock surface and its elevation as encountered at the locations of Boreholes 06-2 and 06-3; bedrock was confirmed by coring at least 3 m in both of these boreholes.

Borehole Location	*Borehole Number*	*Ground Surface Elevation (m)*	*Depth to Bedrock (m)*	*Bedrock Surface Elevation (m)*
West Abutment	06-2	82.7	13.4	69.3
East Abutment	06-3	82.5	11.5	71.0

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

The limestone bedrock at the site is a member of the Gull River Formation; it is slightly weathered to fresh, thinly to medium-bedded, greyish green, and medium strong to strong. The Rock Quality Designation (RQD) values measured on the bedrock core samples recovered from Boreholes 06-2 and 06-3 range from about 85 to 100 per cent, indicating that the bedrock is of good to excellent quality. The discontinuities observed in the rock core are typically horizontal, associated with the bedding planes.

Unconfined compression strength (UCS) tests and point load strength tests were performed on selected samples of the rock core from Boreholes 06-2 and 06-3. The following table summarizes the UCS test results and the approximated UCS as obtained from correlation with the diametral and axial point load strength tests.

Borehole Number	Elevation (m)	Bedrock Type	Test Type	Is Axial (MPa)	Is Diametral (MPa)	Is_{50} (MPa)	UCS (MPa)
06-2	69.1	Limestone	Diametral	-	6.67	5.55	128*
	68.9	Limestone	Axial	3.53	-	3.81	88*
	68.7	Limestone	UCS	-	-	-	108
	68.1	Shale/Dolomite	Diametral	-	1.33	1.29	30*
	68.0	Shale/Dolomite	Diametral	-	2.18	2.12	49*
	67.8	Limestone	Axial	2.88	-	3.10	71*
06-3	71.0	Limestone	Axial	4.27	-	4.46	103*
	70.9	Limestone	UCS	-	-	-	58
	70.5	Limestone	Diametral	-	4.72	4.60	106*
	69.9	Limestone	Diametral	-	4.84	4.72	109*
	69.3	Limestone	Axial	3.28	-	3.44	79*

* The UCS values have been approximated using Is_{50} x 23, from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

The UCS and point load strength test results summarized in the above table indicate that the limestone is classified as a strong rock, while the interbedded shale/dolomite layers are a medium strong rock.

4.3 Groundwater Conditions

A standpipe piezometer was installed and screened within the clay deposit in Borehole 06-1; details of the piezometer installation are shown on the borehole record following the text of this report. The water level measurements in the piezometer in Borehole 06-1 are summarized in the table below.

Depth to Groundwater	Groundwater Level Elevation	Date
0.2 m below ground surface	81.7 m	May 9, 2006
0.3 m above ground surface	82.2 m	January 30, 2007

Based on the above measurements, the groundwater level is generally near ground surface and was slightly artesian in January 2007. The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt; the water level is expected to be higher during the spring season.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Brian Lapos, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of the report.

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PART B

FOUNDATION DESIGN REPORT
DIVISION STREET OVERPASS WIDENING
HIGHWAY 401 WIDENING FROM WEST OF
SYDENHAM ROAD TO WEST OF MONTREAL STREET
KINGSTON, ONTARIO
W.P. 77-99-01

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed widening of the existing single-span overpass that carries Highway 401 over Division Street in Kingston, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

The existing single-span Division Street overpass was constructed in approximately 1954. Based on discussions between MRC and MTO, it is understood that no design information or construction records are available regarding the foundations, except what is shown on Drawing 56388, dated November 5, 1953, which indicates that the abutments are supported on "timber piles" (assumed to be timber friction piles); this drawing does not provide information about the pile diameter or tip elevation. Visual observations by senior Golder personnel of the existing structure do not indicate any settlement-related distress to the structure.

The existing overpass is to be widened approximately 7 m northward, with associated widening of the Highway 401 approach embankments. The widening will match the existing Highway 401 grade, requiring placement of up to 6 m to 6.5 m of new fill on top of the existing embankment side slopes.

As discussed further in Section 6.8, approximately 15 mm to 20 mm of settlement is predicted under the widened embankment loading (assuming the use of conventional earth or granular fill) due to settlement of the clay to clayey silt deposit that underlies the site. This magnitude of settlement assumes that the organic-containing surficial deposit is subexcavated and replaced with compacted Granular "B" fill. If the organic-containing material was left in place, it is predicted that approximately 20 mm to 25 mm of additional settlement would occur, for a total settlement of approximately 35 mm to 45 mm; in addition, ongoing decomposition of the organic material would occur, which would be expected to affect the long-term performance of the approach embankments. Approximately 1.5 m to 2.5 m of subexcavation will be required within the footprint of the approach embankment widening, as outlined in Section 6.8.

Since the existing structure is supported on deep foundations (timber friction piles), it is recommended that deep foundations be adopted for the structure widening to minimize the differential settlement between the new widening and the existing structure; shallow foundations are not recommended based on the predicted magnitude of settlement. Steel H-piles, driven to found on the limestone bedrock, are considered to be the most practical and cost-effective foundation option for the abutment widening and associated wingwalls. Caissons supported on the bedrock could also be considered, although downdrag loads would need to be taken into account for caisson foundation design (since caissons are much stiffer than steel H-piles); in addition, as discussed further in Section 6.5, it would be necessary to socket the caissons into the limestone bedrock due to the undulating/sloping bedrock surface and the presence of water-bearing silty sand till above the bedrock. Temporary liners would also be required during caisson installation to minimize ground disturbance.

From a foundations perspective, either integral or semi-integral abutments on deep foundations could be considered for this site. Abutments on shallow foundations located within a retained/reinforced soil system (RSS) wall structure are not considered suitable given the predicted magnitude of settlement for the grade-supported RSS system.

A brief discussion of geotechnical considerations and recommendations regarding shallow foundations is provided in Section 6.3. Geotechnical recommendations for design of steel H-pile and caisson foundations for support of the abutment widenings are presented in Sections 6.4 and 6.5, respectively. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with these foundation options is presented in Table 1 following the text of this report.

6.3 Shallow Foundations

As noted in Section 6.2, shallow foundations are not recommended for support of the proposed abutment widenings due to the presence of compressible clay to clayey silt at this site and, which would result in differential settlement of the widened sections relative to the existing pile-supported abutments.

Spread footings founded on the stiff to very stiff clay to clayey silt, at or below Elevation 81 m, would have to be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 150 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 100 kPa (for 25 mm of settlement); these resistance values are insufficient for support of the abutment widenings.

To achieve a higher capacity, the footings for the extensions would have to be located at or below Elevation 77 m, which would require a relatively deep excavation and a high abutment wall. Alternatively, footings for the widening could be founded on a compacted Granular “A” pad, placed following subexcavation of the clayey soils down to Elevation 77 m, using a factored geotechnical resistance at ULS of 350 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 175 kPa. However, even with these resistances, there will still be differential settlement of widened footings relative to the existing pile-supported bridge and, therefore, the use of shallow foundations for the widening is not considered feasible.

6.4 Steel H-Pile Foundations

It is recommended that the widened abutments be supported on steel H-piles driven to found on the limestone bedrock. For design, the following pile tip levels may be assumed based on the borehole results; where battered piles are used, it is noted that the bedrock surface is sloping/undulating and the estimated pile tip elevation and pile length can be determined using the interpreted stratigraphic profile on Drawing 1.

Foundation Element	*Borehole Number*	*Approximate Pile Tip Elevation*
West Abutment	06-2	69.3 m
East Abutment	06-3	71.0 m

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the glaciolacustrine clay deposit and the glacial till, and to the potential for deflection of the piles along the sloping/undulating bedrock surface. Based on these considerations, vertically driven piles should be equipped with flange reinforcement (driving shoes) as per SS103-12. Any battered piles should be equipped with suitable driving points (such as Titus standard bearing points or equivalent) to ensure adequate seating of the piles on the bedrock. If there are relatively few vertical piles, standard bearing points can be used for all of the piles since this may be more practicable for contractual purposes.

6.4.1 Axial Geotechnical Resistance

For HP 310x110 piles driven to found on the bedrock, a factored axial geotechnical resistance at ULS of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The axial geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial geotechnical resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

An HP 310x125 pile section could also be considered as an alternative to the above pile section, if required to optimize the design to improve the pile spacing for the widening. In this case, a factored axial geotechnical resistance at ULS of 2,300 kN may be assumed for HP 310x125 piles driven to found on the limestone bedrock. Larger/heavier piles, such as HP 310x132, are produced only at a specific mill and as specific orders, and have not been considered further due to availability and higher cost implications.

Downdrag loading need not be taken into account for the new steel H-piles, as a differential movement of less than 10 mm will occur between the elastic shortening of the new piles under the foundation loading, and the consolidation/compression of the subsoils under the widened embankment loading (as discussed further in Section 6.8). The settlement of the clay soils at the site will, however, affect the existing timber friction piles supporting the northern portion of the existing abutments. It is predicted that the two northern-most timber piles could experience up to 5 mm to 10 mm of settlement as a result of the widened embankment loading. To ensure that the existing overpass structure will not experience this magnitude of settlement, based on discussions with MRC's structural engineers, it is recommended that the widening be rigidly connected to the existing structure such that any downdrag loads on the piles at the north end of the existing pile caps are transferred to the new pile cap.

The Contract Drawings should indicate that pile installation should be in accordance with SP903S01, include the note "Piles to be driven to bedrock", and indicate that the piles should be equipped with driving shoes or rock points, as discussed above.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; all of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

6.4.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. In the case of battered piles, precautions during driving are necessary in some situations (such as for specific soil/bedrock conditions/pile lengths and where the batter is shallower than 6 vertical to 1 horizontal) to ensure that the piles do not deflect along the bedrock surface even with relatively flat-lying bedrock. It is recommended that the pile batter be restricted to 3V:1H or steeper.

If vertical piles are used (i.e. if an integral abutment structure is adopted for the widening), the resistance to lateral loading will have to be derived from the soil in front of the piles. The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equation given below, as described by Terzaghi (1955) and Davisson (1970) as outlined in the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where
k_h is the coefficient of horizontal subgrade reaction (MPa/m);
n_h is the constant of subgrade reaction (MPa/m);
z is the depth (m); and
B is the pile diameter (m).

For cohesive soils:

$$k_h = \frac{67s_u}{B}$$

where k_h is the coefficient of horizontal subgrade reaction (kPa/m); s_u is the undrained shear strength of the soil (kPa); and B is the pile diameter (m).

The following values of n_h and s_u may be assumed in the structural analyses:

Soil Unit	*Elevation*	n_h	s_u
West abutment:			
Compacted granular fill	Above 80 m	5 MPa/m	–
Firm clay	80 m – 78 m	–	40 kPa
Stiff to very stiff clay	78 m – 71 m	–	100 kPa
Hard/very dense till	71 m – 69.3 m	10 MPa/m	–
East abutment:			
Compacted granular fill	Above 80.5 m	5 MPa/m	–
Stiff clay	80.5 m – 79.5 m	–	60 kPa
Firm clay	79.5 m – 77.5 m	–	40 kPa
Stiff to very stiff clay to clayey silt	77.5 m – 74 m	–	100 kPa
Stiff to hard/compact to very dense till	74 m – 71 m	10 MPa/m	–

A maximum lateral resistance of 160 kN at ULS, and a maximum lateral resistance of 65 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles. These values are based on the "Assessed Horizontal Passive Resistance Values for Various Pile Types" provided in Table C6.8.7.1(a) of the *Commentary* to the *CHBDC*.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R, as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	*Subgrade Reaction Reduction Factor*
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

6.4.3 Frost Protection

The pile caps should be provided with a minimum of 1.5 m of soil cover for frost protection.

6.5 Caissons

As an alternative to steel H-piles, caissons could be considered for support of the abutment widenings. However, caissons will be more difficult to construct since it will be necessary, given the presence of the borderline cohesionless till above the bedrock and the slightly sloping/undulating bedrock surface, to use a temporary liner during construction and to socket the caissons at least 0.5 m into the bedrock. The limestone bedrock is moderately strong to strong, and so the sockets would have to be advanced into the rock by churn drilling or rock coring supplemented by down-hole hammer. Temporary liners would be required to minimize loss of the silty sand till soils above the bedrock, as well as to reduce the possibility for "flow" of the medium-sensitive clay soils if left unsupported during caisson installation.

An alternative to socketing the caissons into the bedrock, dowels could be installed into the bedrock at the caisson base, and the caisson formed over top of the dowels. This option could eliminate the requirement for rock coring/churn drilling, but there is still potential for difficulty associated with "sealing" the temporary liner onto the sloping bedrock surface in order to maintain a dry and clean caisson hole to permit installation of the dowels. Based on this, it is considered that this option is not practicable.

6.5.1 Axial Geotechnical Resistance

Caissons socketed approximately 0.5 m into the bedrock should be designed based on end-bearing resistance, using a factored axial geotechnical resistance at ULS of 7 MPa; for a 1.5 m diameter caisson, this would equate to a factored axial geotechnical resistance at ULS of 12,000 kN. Serviceability Limit States (SLS) resistances do not apply to caissons founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

The widening of the approach embankments will raise the effective stress level in the grey clay deposit at depth close to (but not exceeding) its estimated preconsolidation pressure, and will induce approximately 15 mm to 20 mm of settlement in the clay deposit. Although downdrag loads can be neglected for steel H-piles, concrete caissons are relatively stiffer and the elastic shortening of the caissons will be significantly less than for steel H-piles. Therefore, the differential movements are expected to be sufficient to generate downdrag forces on the caissons. In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. The neutral plane used in the analyses was assumed to be at the underside of the grey clay deposit. Based on the above, the unfactored downdrag load acting over the length of a 1.5 m diameter caisson within the clay deposit is estimated to be 1,500 kN. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*.

6.5.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.4.2.

6.5.3 Frost Protection

The pile caps should be provided with a minimum of 1.5 m of soil cover for frost protection.

6.6 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.0, consistent with Soil Profile Type I. The following table provides the applicable seismic design parameters in accordance with Table A3.1.7 of the *CHBDC* for the Kingston area.

Velocity Related Seismic Zone, Z_v	1
Zonal Velocity Ratio, V	0.05
Acceleration Related Seismic Zone, Z_a	2
Zonal Acceleration Ratio, A	0.1

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill at ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with MTO's Special Provision SP105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision SP105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular "A"	**Granular "B" Type II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows:

  - Rotation of approximately 0.002 about the base of a vertical wall;
  - Horizontal translation of 0.001 times the height of the wall; or
  - A combination of both.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Section 4.4.4.1 of the *Commentary to the CHBDC*, this site is located in Seismic Performance Zone 2. The site-specific zonal acceleration ratio, A, for Kingston is 0.1. The peak horizontal ground acceleration, g, for 10% probability of exceedance in 50 years is 0.11g.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding (i.e. the abutment walls for this structure), the horizontal seismic coefficient, k_h, used in the calculation of the seismic lateral earth pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.15$). For structures which allow lateral yielding (i.e. the wing walls for this structure), k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.05$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

Wall Type	*Case I*	*Case II*
Yielding wall	0.35	0.31
Non-yielding wall	0.44	0.40

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.1. This corresponds to displacements of up to approximately 25 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a\ \gamma\ d + (K_{AE} - K_a)\ \gamma\ (H\text{-}d)$$

where

$\sigma_h(d)$	is the lateral earth pressure at depth, d, (kPa)
K_a	is the static active earth pressure coefficient;
K_{AE}	is the seismic active earth pressure coefficient;
γ	is the unit weight of the backfill soil (kN/m^3), as given previously;
d	is the depth below the top of the wall (m); and
H	is the total height of the wall (m).

6.8 Approach Embankment Design and Construction

The natural ground surface immediately to the north of the existing Highway 401 embankment is relatively flat, at about Elevation 82 m to 83 m. In the northward widening of the Highway 401 approach embankments, the new embankment fill will be placed to match the existing grade (approximately Elevation 88.5 m at the west approach, and 89 m at the east approach). The northward widening of the west and east approach embankments will, therefore, require placement of approximately 6 m to 6.5 m of fill on top of the existing embankment side slopes.

6.8.1 Subgrade Preparation and Approach Embankment Construction

In order to improve embankment performance and minimize differential settlement between the existing and widened portions of the approach embankments, it is recommended that all topsoil, organic silt and organic-containing surficial clayey silt to clay be stripped from beneath the approach embankment footprints (including topsoil on the existing Highway 401 embankment side slopes), prior to embankment fill placement. Subexcavation of organic-containing soil that may be present under the existing Highway 401 embankment toe should be carried out in short sections perpendicular to the Highway 401 alignment, with the base of the excavation/trench not wider than 3 m at any time. Excavation and backfilling operations should be carried out simultaneously such that the excavation is not left open for more than 3 m in length at any given time.

The following stripping depths and subgrade elevations are recommended based on the results from Boreholes 06-1 to 06-4:

Location	*Depth of Stripping*	*Subgrade Elevation*
West approach embankment	Approx. 1.5 m to 2.5 m	80.0 m to 80.5 m
East approach embankment	Approx. 2.0 m	80.5 m

This subgrade should be inspected following subexcavation to ensure that all organic-containing soils have been fully removed, then the subexcavated area should be replaced with Granular "B" backfill that is placed and compacted in accordance with the requirements of MTO's Special Provision SP105S10. It is noted that, following stripping of the topsoil and organic-containing surficial soils, the clayey subgrade will be susceptible to disturbance as a result of equipment travelling over the surface. It is recommended that an Operational Constraint be included in the Contract Documents to restrict travel over the exposed subgrade; further discussion on this aspect is provided in Section 6.9.

As an alternative to stripping of the organic-containing surficial deposit below the widening area, the use of deep soil mixing or rammed aggregate piers could be considered to improve the subgrade performance in the embankment widening area. The mobilization costs for deep soil mixing would be high; discussions with a contractor have indicated that it would not be practical to mobilize equipment to the site for the relatively limited improvement works required for the Division Street overpass approach embankment widening, and so this option is not considered to be feasible. Rammed aggregate piers, which can be installed to a maximum of about 7.5 m depth, are considered feasible for this site; discussions with a specialist contractor have indicated that for the depth of subexcavation required at this site, rammed aggregate piers would be expected to be cost-competitive, and would have the added advantage of avoiding subexcavation under the toe of the existing embankment. Further design would be required if this option is pursued.

The embankment fill for the widening should be placed and compacted in accordance with MTO's Special Provision SP105S10. The use of granular fill is recommended over the use of cohesive fill for the widening, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. Benching of the existing Highway 401 embankment side slopes should be carried out to "key in" the new fill materials in accordance with OPSD 208.010.

To reduce surface water erosion on the new or widened embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.8.2 Approach Embankment Stability

Static and seismic slope stability analyses of the embankments were carried out with the commercially available program SLOPE-W (produced by Geo-Slope International Ltd.), using the soil parameters given in the following table.

Soil Deposit	*Bulk Unit Weight*	*Effective Friction Angle*	*Undrained Shear Strength*
Embankment fill (range of parameters assumed for earth and granular fill)	20 – 22 kN/m^3	32° to 35°	–
Firm clay	18 kN/m^3	–	40 kPa
Stiff to very stiff clay	18 kN/m^3	–	100 kPa
Clayey silt to silty sand till	21 kN/m^3	N/A (See note)	

NOTE: Due to the significant difference in shear strength and stress-strain response of the clay versus the underlying stiff to hard/compact to very dense granular till deposit, the critical failure surface for a deep-seated instability of the embankment does not penetrate the till deposit.

The results of the slope stability analyses indicate that the 6 m to 6.5 m high embankment widening with side slopes maintained at 2H:1V will have a factor of safety of about 1.7 against deep-seated slope instability, for the undrained conditions during and immediately after embankment construction, under static loading conditions; an example of the stability analysis results is shown on Figure 7. Under seismic loading conditions with a seismic coefficient (ground acceleration, g) of 0.1, the factor of safety is maintained greater than 1.0. These results assume appropriate subgrade preparation (removal of the surficial organic-containing soils) and proper placement and compaction of embankment fill materials.

6.8.3 Approach Embankment Settlement

Settlement of the approach embankments will occur as a result of compression of the new embankment fill itself, as well as consolidation of the clayey soils underlying the widened approach embankments.

Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the embankment fill itself is expected to be up to about 10 mm. The use of granular fill for the new embankment construction will reduce this magnitude of settlement since the majority of settlement of granular fills will occur during construction, whereas the majority of the settlement of cohesive fill, if used, would occur shortly after construction.

To estimate the magnitude of settlement, analyses were carried out using the commercially-available program Unisettle as well as hand calculations. The settlement of the founding soils has been estimated using the consolidation parameters and elastic deformation moduli given in the table below, based on correlations with the undrained shear strength, Atterberg limits and SPT "N" values. The consolidation parameters for the firm clay are consistent with the oedometer test results for a sample of firm silty clay to clay obtained as part of the Division Street W-N/S Ramp investigation, immediately southeast of the structure site.

Soil Type	*Bulk Unit Weight*	*Elastic Modulus*	P_c'	e_o	C_c	C_r
Embankment fill (range of parameters assumed for earth fill and granular fill)	20-22 kN/m^3	–	–	–	–	–
Firm clay	18 kN/m^3	–	180 kPa	0.9	0.4	0.05
Stiff to very stiff clay	18 kN/m^3	10 – 15 MPa	–	–	–	–
Very stiff to hard / compact to very dense till	21 kN/m^3	40 MPa	–	–	–	–

The additional loading due to the widening of the approach embankments will approach but will not exceed the preconsolidation pressure of the firm portion of the clay deposit. Assuming removal of the organic-containing surficial soil deposit, the total estimated settlement of the clay to clayey silt founding soils under the widened approach embankments will be approximately 15 mm to 20 mm. It is predicted that the two northern-most timber piles supporting the existing abutments will experience approximately 5 mm to 10 mm of settlement; however, based on discussions with MRC's structural engineers and as recommended in Section 6.4, the widening will be rigidly connected to the existing structure resulting in load sharing, and so the existing overpass structure will not experience this magnitude of settlement.

Based on Terzaghi's one-dimensional consolidation theory, it is estimated that 90 per cent of the primary consolidation settlement will be completed within approximately one year following completion of the embankment widening.

6.9 . Design and Construction Considerations

6.9.1 Open-Cut Excavations

Based on the proposed pile cap underside of Elevation 81 m, foundation excavations extending to approximately 1.5 m below the adjacent Division Street grade will be required. Additional excavation will be required to subexcavate the surficial organic silt and clayey silt to clay containing organics from within the footprint of the approach embankment widening. As outlined in Section 6.8, approximately 1.5 m to 2.5 m of subexcavation will be required to remove this organic-rich material; controlled excavation in strips of specified width and length will be required to remove this material from under the existing Highway 401 embankment toe, and a sample Operational Constraint to address these requirements is included in Appendix A.

The excavations will extend through existing fill (associated with both Division Street and the Highway 401 embankment), the surficial deposit and into the clay deposit. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The site soils are classified as Type 3 soils according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes not steeper than 1 horizontal to 1 vertical (1H:1V).

The clay deposit that will be exposed in the pile cap excavations and within the approach embankment widening area will be sensitive to disturbance from ponded water and construction traffic. As discussed in Section 6.8, it is recommended that an Operational Constraint be included in the Contract Documents to restrict travelling over the exposed clayey subgrade in order to minimize such disturbance prior to placement of the Granular "B" backfill. A sample Operational Constraint is included in Appendix A.

6.9.2 Temporary Roadway Protection

Depending on construction staging, temporary roadway protection may be required along Highway 401 and along Division Street to facilitate construction of the abutment widenings. The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19, provided that any buried utilities that may be present adjacent to the excavation(s) can tolerate this magnitude of deformation.

For conceptual planning purposes, it is anticipated that the temporary excavation support system could consist of soldier piles and lagging or interlocking steel sheet piles, with lateral support provided (if necessary) by soil anchors.

6.9.3 Groundwater Control

The hydrostatic level associated with the clay deposit at this site is relatively high, near ground surface. Minor seepage from the clay should be expected. Any granular fill or organic silt materials at the site, including granular fill placed within the subexcavation areas, should be expected to be water-bearing, with water "perched" on top of the relatively impermeable clay, particularly during wet periods of the year. It is anticipated that the groundwater seepage into the foundation excavations can be adequately controlled by pumping from properly filtered sumps.

6.9.4 Obstructions During Pile Driving and Protection System Installation

It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the Contractor of the presence of cobbles and boulders within the overburden soils, which are glacially derived, as such obstructions may affect the installation of steel H-piles for abutment widenings, or the installation of soldier piles, steel sheeting, anchors and other elements of protection systems. Sample NSSPs are provided in Appendix A.

6.9.5 Vibration Monitoring During Pile Installation

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing overpass structure are maintained below tolerable levels. An NSSP should be included in the Contract Documents for this purpose. A sample NSSP is provided in Appendix A.

A maximum peak particle velocity of 50 mm/s is recommended at the existing overpass structure. The piles further from the existing structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the pile driving criteria for the remaining piles.

6.9.6 Formation of Bedrock Sockets

If caissons are adopted and rock sockets are required, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor that the bedrock is medium strong to strong, which will require socket formation using coring or churn drilling to advance the hole. If this foundation option is adopted, a sample NSSP will be prepared for inclusion in the Contract Documents.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Brian Lapos, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder, with technical input from Mr. Murty Devata, P.Eng., a specialist foundations consultant to Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of the report.

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BML/LCC/MSD/FJH/bml/lcc
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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
DIVISION STREET OVERPASS WIDENING
W.P. 77-99-01

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs
Spread footings supported on native clay to clayey silt	• Not feasible due to differential settlement relative to existing pile-supported structure		• Low bearing resistance • Not compatible with existing structure foundations	• Less expensive than deep foundations, but costs would increase due to subexcavation requirements and Granular "A" requirements
Steel H-pile foundations driven to found on bedrock	• Feasible for support of abutments	• High bearing resistance • Negligible settlement • Faster installation than caissons	• Possibility of encountering cobbles or boulders during pile driving • Care must be taken with driving of battered piles to ensure that the piles do not deflect along the bedrock surface	• Less expensive than caissons
Caissons socketed nominally into bedrock	• Feasible for support of abutments	• High bearing resistance • Negligible settlement	• Temporary liners required to prevent running or flowing of silty sand till above bedrock, and possible disturbance of "remoulded" clay • Possibility of encountering cobbles or boulders during drilled shaft installation • Coring or churn drilling will be required to form socket in medium strong to strong bedrock	• More expensive than steel H-pile option

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u,s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

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LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_C	consistency index $= (w_l - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p/\sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
ll - Parallel To	
⊥ - Perpendicular To	

Form...G.A.-R-3

Golder Associates

RECORD OF BOREHOLE No 06-1

 METRIC

PROJECT 05-1111-031
77-99-01
LOCATION N 4903276.1 ;E 304850.0
ORIGINATED BY DM
DIST ____ HWY 401
BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers
COMPILED BY BL
DATUM Geodetic
DATE February 8, 2006
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.9	GROUND SURFACE				
0.0	Topsoil				
0.2	Silty Clay, trace sand, containing organics Firm Dark brown Moist	1	SS	4	OC=6.8
80.6 / 1.4	Clay, trace sand Stiff to very stiff Grey Moist to wet	2	SS	6	
		3	SS	6	0 5 28 67
		4	SS	6	
		5	SS	2	
		6	SS	3	
75.2 / 6.7	End of Borehole				

Field vane readings near elevation 78: 47, >100.

Notes:

1. Water level in open borehole at 5.6m depth (Elev. 76.3m) upon completion of drilling operations.

2. Water level measured in piezometer on May 9, 2006 at 0.2 m depth (Elev. 81.7 m).

3. Water level measured in piezometer on January 30, 2007 at 0.29 m above ground surface (Elev. 82.2 m).

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (81, 80, 79, 78, 77, 76); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.

+³, ×³: Numbers refer to Sensitivity   O ³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 06-2

 METRIC

PROJECT 05-1111-031			
W.P. 77-99-01	LOCATION N 4903268.1 ;E 304862.0		ORIGINATED BY DM
DIST	HWY 401	BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers	COMPILED BY BL
DATUM Geodetic	DATE February 9, 2006		CHECKED BY LCC

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa / DYNAMIC CONE PENETRATION	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.7	GROUND SURFACE									
0.0	Gravel and silty organics (FILL) Grey Moist									
82.1 0.6	Clay, trace sand, containing organics Firm to stiff Grey Moist					82				
		1	SS	10						
		2	SS	6		81		o		OC=6.6
80.2 2.5	Clay, trace sand Firm to very stiff Grey Moist becoming wet below Elevation 77.0m	3	SS	8		80				
		4	SS	4				├─o┤		
						79	+ 2.5 + 1.8			
		5	SS	3		78		o		
							+ >100			
					∇	77	+ >100			
		6	SS	8		76				
							+ >100			
		7	SS	4		75		├o──┤		0 1 14 85
							+ >100			
						74				
		8	SS	6		73		o		
72.7										

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P; NATURAL MOISTURE CONTENT W; LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 06-2

 METRIC

PROJECT 05-1111-031

W.P. 77-99-01 LOCATION N 4903268.1 ;E 304862.0 ORIGINATED BY DM

DIST ______ HWY 401 BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers COMPILED BY BL

DATUM Geodetic DATE February 9, 2006 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
10.0	Clay, trace sand Firm to very stiff Grey Moist to wet							+ >100			
							72				
			9	SS	2						
71.1 11.6	Clayey Silt with sand, some gravel to Silty Sand, some gravel, trace clay (TILL) Hard/Very dense Grey Wet						71				
			10	SS	54				⊣○⊢		
							70				
69.3 13.4	Limestone (BEDROCK) Bedrock cored between 13.4 and 16.8m depth. For bedrock coring details, refer to Record of Drillhole 06-2.						69				
							68				
							67				
							66				
65.9 16.8	End of Borehole Note: 1. Water level in open borehole at 5.7m depth (Elevation 77m) upon completion of drilling operations.										

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

MIS-MTO 001 05111031.GPJ GAL-MISS.GDT 22/11/06

PROJECT: 05-1111-031

RECORD OF DRILLHOLE: 06-2

LOCATION: N 4903268.1 ;E 304862.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 9, 2006

DRILL RIG: CME-55

DRILLING CONTRACTOR: Marathon Drilling Ltd.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate, BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage, PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular, PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	CONTINUED FROM BOREHOLE 06-2	69.28			
	Limestone (BEDROCK) Slightly weathered to fresh Medium strong to strong Thinly to medium-bedded Greyish-green	13.40	1		
14				BD, PL, Ro	UCS = 108 Mpa
			2		
15				BR	
16			3	BD, PL, Ro	
	Dolomite from 16.55m to bottom			BD, IR, Ro	
		65.88		BD, PL, Ro	
	End of Drillhole	16.80			
17					
18					
19					
20					
21					
22					
23					

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: DM

CHECKED: LCC

RECORD OF BOREHOLE No 06-3

 METRIC

PROJECT 05-1111-031
W.P. 77-99-01 LOCATION N 4903262.9 ;E 304882.6 ORIGINATED BY DM
DIST ____ HWY 401 BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers COMPILED BY BL
DATUM Geodetic DATE February 14, 2006 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.5	GROUND SURFACE									
0.0	Silty sand (FILL) Grey Wet					82				
81.8 / 0.8	Clayey Silt, trace sand, containing organics and decayed wood Firm Dark grey Moist	1	SS	5		81				
		2	SS	5	∇					
80.7 / 1.8	Clay, trace sand Firm to very stiff Grey Moist	3	SS	12		80				
		4	SS	5		79				
		5	SS	3		78				
		6	SS	3						
						77	+ >100			
		7	SS	4		76				
	Becoming wet below 6.7 m depth						+ >100			
75.0 / 7.5	Clayey Silt, trace sand Very stiff Grey Wet	8	SS	2		75				
						74	+ >100			
74.0 / 8.5	Clayey Silt with sand, some gravel to Silty Sand, some gravel, trace clay (TILL) Stiff to Hard/Compact to very dense Grey Wet	9	SS	19		73				

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity ○ ³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 06-3

 METRIC

PROJECT 05-1111-031

W.P. 77-99-01 LOCATION N 4903262.9 ;E 304882.6 ORIGINATED BY DM

DIST HWY 401 BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers COMPILED BY BL

DATUM Geodetic DATE February 14, 2006 CHECKED BY LCC

— CONTINUED FROM PREVIOUS PAGE —

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Clayey Silt with sand, some gravel to Silty Sand, some gravel, trace clay (TILL) Stiff to Hard/Compact to very dense Grey Wet					72					
		10	SS	72				19	62	11	8
71.0 / 11.5	Limestone (BEDROCK) Bedrock cored between 11.5 and 14.6m depth. For bedrock coring details, refer to Record of Drillhole 06-3.					71					
						70					
						69					
67.9 / 14.6	End of Borehole Note: 1. Water level in open borehole at 1.6m depth (Elevation 80.9m) upon completion of drilling operations.					68					

Column headings: SOIL PROFILE; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); REMARKS & GRAIN SIZE DISTRIBUTION (%); STRAT PLOT

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: 06-3

PROJECT: 05-1111-031

SHEET 1 OF 1

LOCATION: N 4903262.9 ;E 304882.6

DRILLING DATE: February 14, 2006

DATUM: Geodetic

DRILL RIG: CME-55

INCLINATION: -90°   AZIMUTH: —

DRILLING CONTRACTOR: Marathon Drilling Ltd.

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate, BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage, PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular, PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	CONTINUED FROM BOREHOLE 06-3	71.01			
	Limestone (BEDROCK) Slightly weathered to fresh Medium strong to strong Thinly to medium-bedded Greyish-green	11.50		BR JN, PL, SM PL, SM	UCS = 58 Mpa
12					
	Minor SILTSTONE from 13.1m to 13.2m.		1	BD, PL, Ro	
				BD, PL, Ro JN, UN, Ro BD, PL, Ro	
13					
			2		
14					
		67.91			
	End of Drillhole	14.60			
15					
16					
17					
18					
19					
20					
21					

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: DM

CHECKED: LCC

Golder Associates

RECORD OF BOREHOLE No 06-4

 METRIC

PROJECT 05-1111-031			
W.P. 77-99-01	LOCATION N 4903269.7 ;E 304894.7	ORIGINATED BY DM	
DIST	HWY 401	BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers	COMPILED BY BL
DATUM Geodetic	DATE February 15, 2006	CHECKED BY LCC	

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.8	GROUND SURFACE										
0.0	Topsoil										
0.2	Organic Silt containing rootlets Soft to firm Dark grey/black Moist						82				
			1	SS	5						
			2	SS	2		81				
80.8 2.0	Clay, trace sand Firm to very stiff Grey Moist					▽					
								>100			
							80				
			3	TO	PH						
							79	>100			
			4	SS	7		78				
								>100			
							77				
			5	SS	4						
76.1 6.7	End of Borehole Note: 1. Water level in open borehole at 2.1 m depth (Elevation 80.7m) upon completion of drilling operations.										

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 07-21

 METRIC

PROJECT 05-1111-031

77-99-01

LOCATION N 4903260.3 ;E 304822.3

ORIGINATED BY DM

DIST ____ HWY 401

BOREHOLE TYPE Track-Mounted C.M.E. 75, 200mm O.D. Hollow Stem Augers

COMPILED BY LCC

DATUM Geodetic

DATE January 22, 2007

CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
88.0	GROUND SURFACE									
0.0	ASPHALT									
0.3	Sand and gravel (FILL) Compact Grey									
87.2 / 0.8	Sand, trace gravel (FILL) Compact Brown Moist	1	SS	26	87					
86.6 / 1.4	Sand and gravel, trace silt (FILL) Compact Brown Moist	2	SS	20	86	o				
85.9 / 2.3	Clayey silt, trace sand and gravel (FILL) Very stiff Brown Moist	3	SS	47						
	Gravel and cobbles, some sand (ROCK FILL) Loose to dense Grey Moist	4	SS	12	85					
		5	SS	5	84	o				
		6	SS	4	83					
		7	SS	24						
82.1 / 6.0	TOPSOIL Clayey silt with to some sand, trace gravel (FILL) Stiff to very stiff Grey-brown Moist	8	SS	10	82	o				
		9	SS	16	81	o	4	23	41	32
80.5 / 7.5	SILTY CLAY, trace gravel Stiff Brown Moist	10	SS	14	80	Wp–w–WL				
79.9 / 8.1	End of Borehole									

Notes:

1. Borehole dry upon completion of drilling.

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 07-22

 METRIC

PROJECT 05-1111-031
77-99-01
LOCATION N 4903249.0 ;E 304936.8
ORIGINATED BY DM
DIST HWY 401
BOREHOLE TYPE Track-Mounted C.M.E. 75, 200mm O.D. Hollow Stem Augers
COMPILED BY LCC
DATUM Geodetic
DATE January 11, 2007
CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
89.5	GROUND SURFACE						
0.0	ASPHALT						
0.1	Sand and gravel, trace silt, containing cobbles (FILL) Loose to very dense Grey Moist	1	SS	79		89	
		2	SS	6		88	
		3	SS	13		87	
		4	SS	8		86	
85.8 / 3.7	Clayey silt, some sand, trace gravel (FILL) Soft Brown Moist	5	SS	3			
85.1 / 4.4	Gravel and cobbles, some sand (ROCK FILL) Very dense	~~6~~	~~SS~~	~~50/0.01~~		85	
84.3 / 5.2	Clayey silt, some sand, trace gravel (FILL) Stiff Brown Moist	7	SS	15		84	4 17 51 28
83.6 / 5.9	SILTY CLAY, trace sand Very stiff Brown Moist	8	SS	17		83	
83.0 / 6.6	End of Borehole						

Notes:

1. Borehole dry upon completion of drilling.

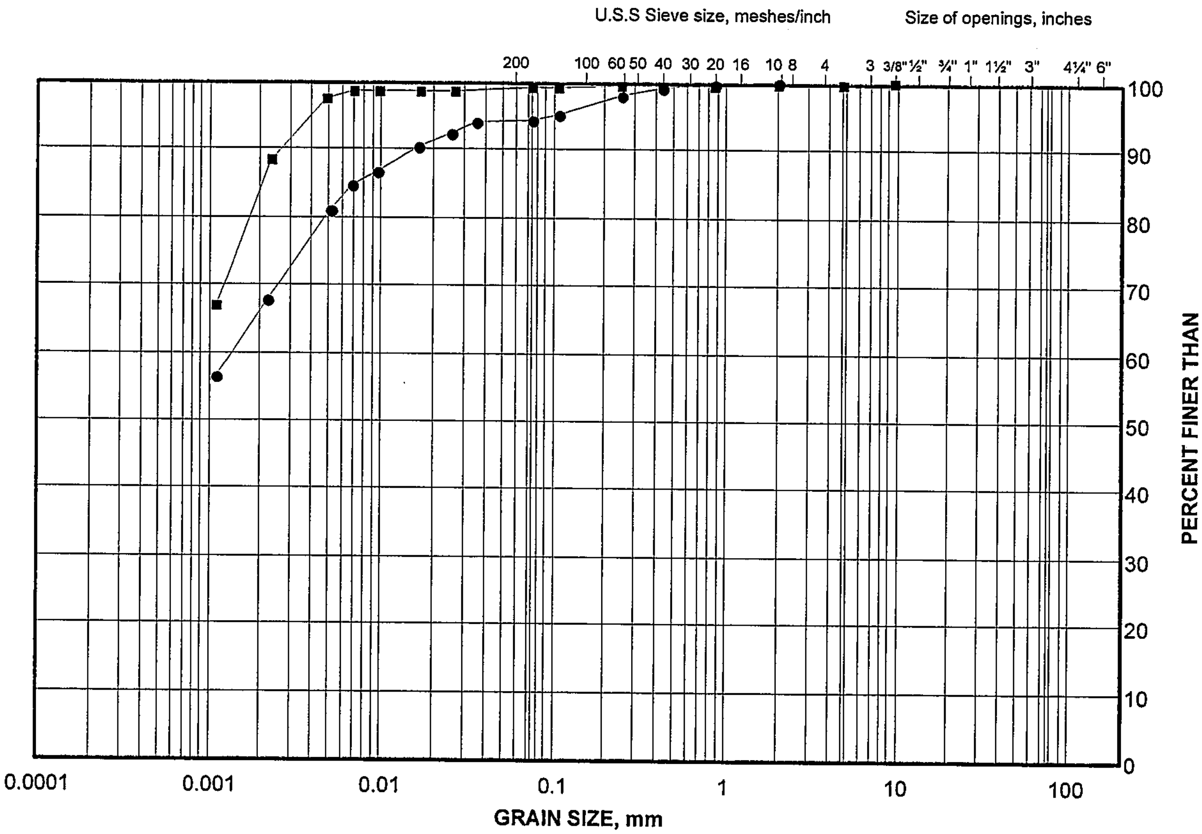
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75
UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clay

FIGURE 3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	06-1	3	79.6
■	06-2	7	75.1

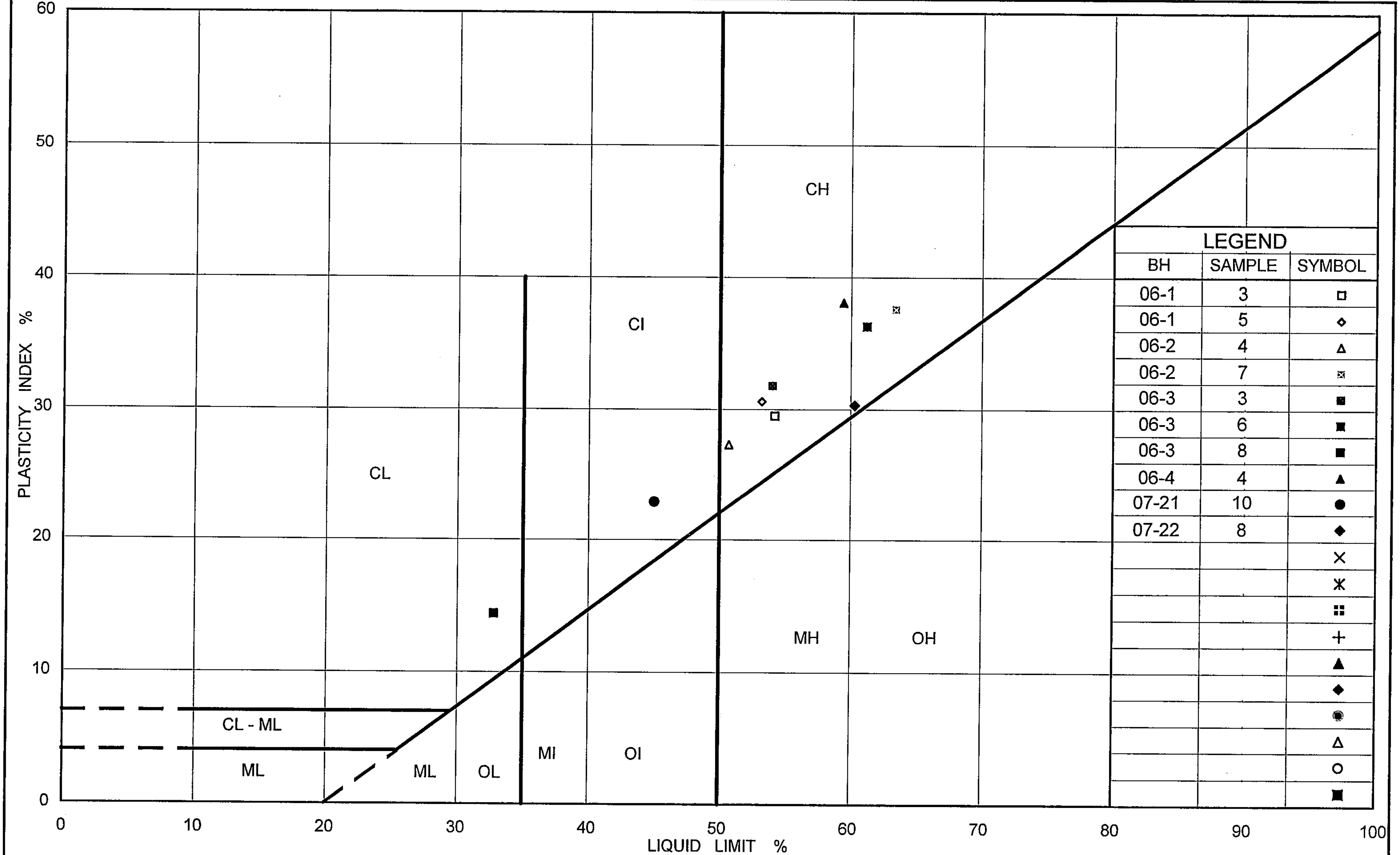
Project Number: 05-1111-031

Checked By: [signature]

Golder Associates

Date: 2-Nov-07

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clay to Clayey Silt

Figure No. 4

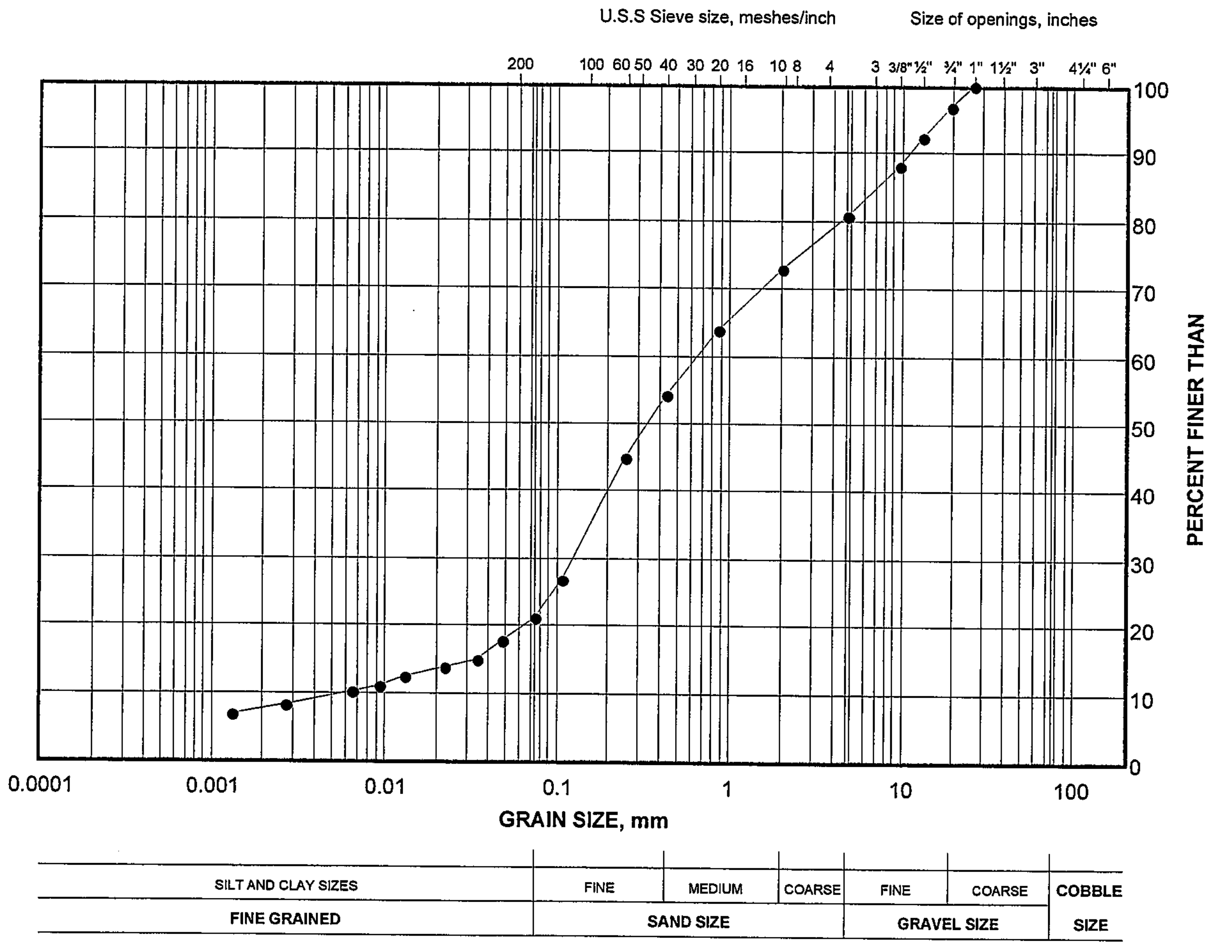
Project No. 05-1111-031

Date: November, 2007

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till to Silty Sand Till

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-3	10	71.8

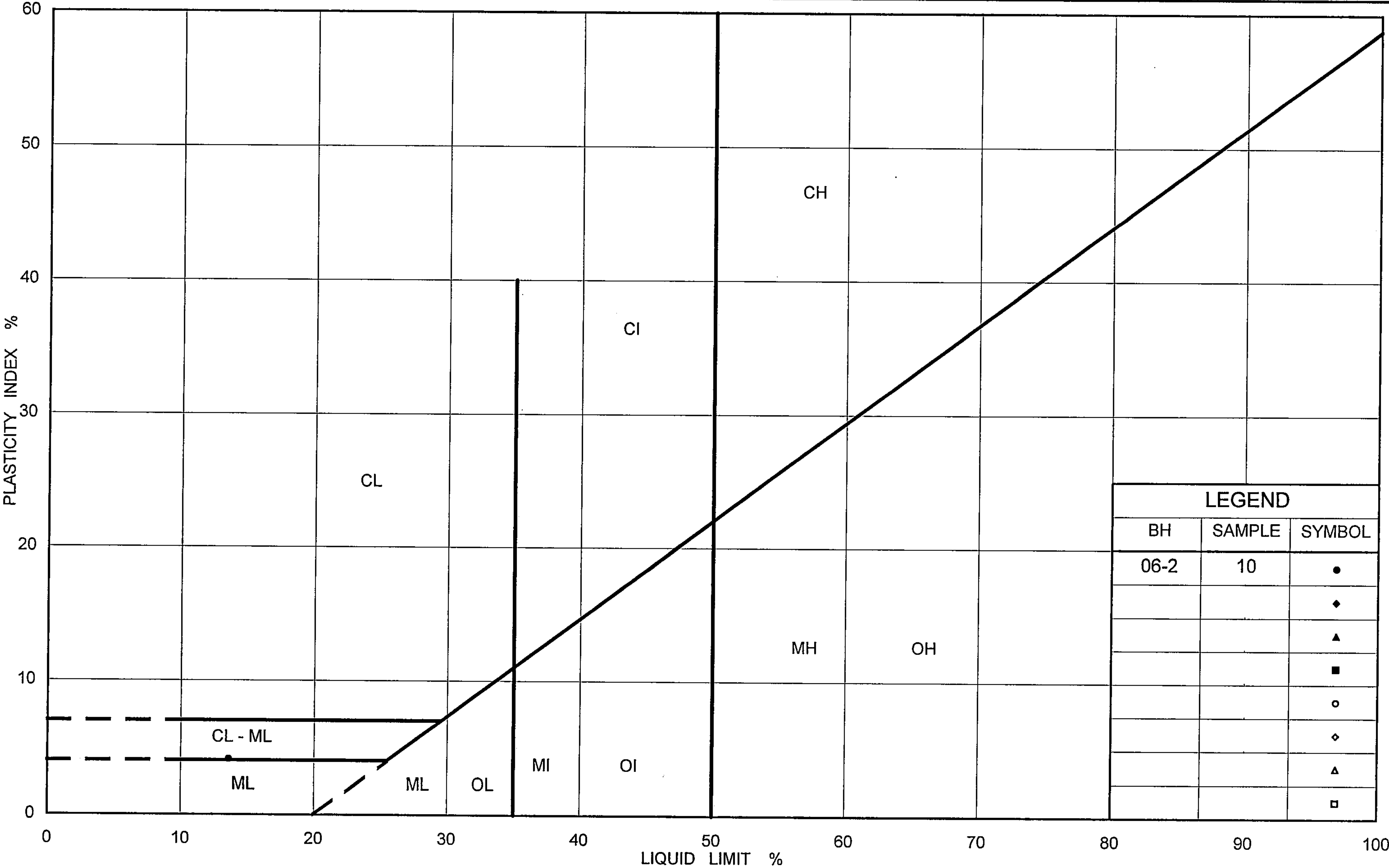
Project Number: 05-1111-031

Checked By: [signature]

Golder Associates

Date: 2-Nov-07

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
06-2	10	●
		◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Sand Till

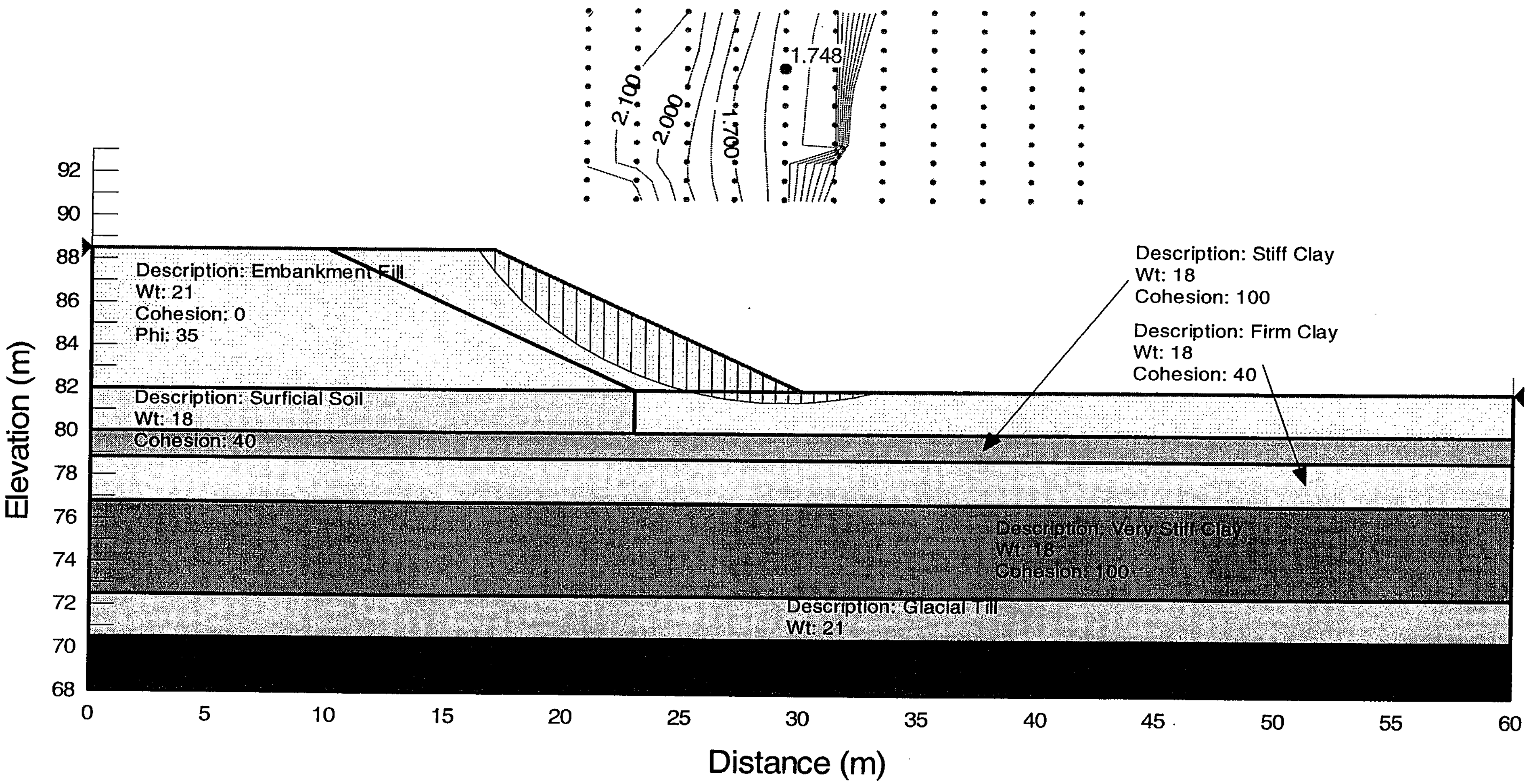
Figure No. 6

Project No. 05-1111-031

Date: November, 2007

Slope Stability Assessment
Division Street Overpass Widening

FIGURE 7



Date:	November 2007		Drawn:	BML/JB
Project:	05-1111-031	**Golder Associates**	Checked:	LCC/JMAC

APPENDIX A

OPERATIONAL CONSTRAINTS
AND
NON-STANDARD SPECIAL PROVISIONS

OPERATIONAL CONSTRAINT

Special Provision

Subexcavation of Organic-Containing Surficial Soils at Division Street Overpass Approach Embankment Widening

This special provision outlines the procedure to be used for excavation of the organic silt and organic-containing clayey silt to clay deposits within the limits of the northward widening for the Division Street overpass approach embankments.

Removal of the organic-containing soils shall be in accordance with OPSD 203.020 except as noted herein.

Removal of the organics/clay shall be carried out in short sections perpendicular to the Highway 401 alignment, with the base of the excavation/trench not wider than 3 m at any time.

Excavation and backfilling operations shall be carried out simultaneously in a manner that the excavation is not left open for more than 3 m in length at any given time.

The Contractor shall maintain the operation of the existing highway during excavation and backfilling operations, including and not limited to traffic control, regrading and asphalt padding.

Basis of Payment

Payment for the Contractor to provide the above requirements, including all equipment, labour and materials shall be deemed to be included in the contract bid price for the various tender items.

OPERATIONAL CONSTRAINT

Special Provision

Protection of Subgrade Soils at Division Street Overpass Site

In order to limit disturbance to the clayey subgrade soils that will be exposed within the embankment footprints at the Division Street overpass site, following stripping of the surficial organic-containing deposit:

- Construction equipment shall not travel over the clayey subgrade soils until Granular "B" Type II fill is placed on top of the clayey subgrade. The Granular "B" fill should be end-dumped and spread with a light bulldozer to a minimum thickness of 0.6 m before any truck or heavy equipment traffic is permitted to work over the subexcavation area for construction of the embankment widenings.

BOULDERS/OBSTRUCTIONS DURING PILE INSTALLATION - Item No.

Special Provision

The soils at the site are glacially-derived and should be expected to contain cobbles and boulders. Appropriate equipment and procedures will be required to penetrate obstructions (cobbles and boulders) that are encountered during pile driving.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

BOULDERS/OBSTRUCTIONS DURING INSTALLATION OF PROTECTION SYSTEMS - Item No.

Special Provision

The soils at the site are glacially-derived and should be expected to contain cobbles and boulders. Appropriate equipment and procedures will be required to penetrate obstructions (cobbles and boulders) that are encountered during the installation of soldier piles, anchors/tie-backs, or other elements of the protection systems.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during the piling installation works for the widening of the existing Division Street overpass structure.

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

Submission Requirements

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

Qualifications of vibrations monitoring specialist.
Proposed instrumentation.
Proposed location of instruments on existing Division Street overpass structure.
Proposed frequency of readings.
Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

Pile Driving and Monitoring

The vibration monitoring equipment shall be placed on the existing overpass structure, as close as possible to the piling works. Pile driving shall commence with the pile furthest away from the existing structure for each widening area. The Contractor shall take readings on the existing structure during driving of each pile and during seating of the piles on the bedrock.

The vibrations measured on the existing structure shall not exceed 50 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations at the existing structure are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION